Monitoring rock slopes at Loktak project India

L'auscultation des pentes rocheuses au projet Loktak en Inde

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SYNOPSIS: The slopes at Loktak hydro-electric project located in the north eastern Hymalayan region have been failing ever since the construction started. Several remedial measures were taken up including monitoring their performence with the help of inclinometer. Significant movement has been recorded over a period of last two years. The problem of slope stability and its monitoring are presented in this paper.

1 INTRODUCTION

The Loktak Hydro-Electric Project is located in the North-Eastern part of the Himalayan region in the State of Manipur, India. It utilises a fall of about 312m by transferring water from Loktak lake to Liematak valley for generating 105 MW of power. The water conductor system includes about 2.31 km. open channel, 1.1 km. cut and cover section, 6.6 km. long head race tunnel (HRT), a 60m high, 9.15m diameter surge shaft, about 0.3 km. steel lined tunnel and three penstock of 1.3 km. length each for three units as shown in Figure 1. During the construction phase, it faced several problems causing delay in its commencing. There were large scale failures of slopes around open channel and penstock area. Methane gas was encountered in the tunnel which accounted for lives of a few people. The project was commissioned in June 1983. This paper deals with some of the slope stability problems, and its monitoring by field instrumentation carried out by the Central Soil and Materials Research Station (CSMRS), New Delhi.

1.1 Power channel

The power channel which leads the water from the lake to the inlet of the tunnel is 2.31 km. long and is designed to carry 60 cubic metres of water per second. The strength of the organic clay through which the channel was aligned, was vary low resulting in frequent slope failures. The failures were characterised by extensive crack development longitudinally with gradual heaving of the channel bed by about 1.5 to 2 metres. Several methods of protection of the slope, including toe stabilization trenches, stone pitching with gravel and filter backing and driving of reamed piles to improve the shear strength characteristics of the clay material were tried, with little success, though. Ultimately, two rows of sheet piles - heavily strutted - had to be driven to retain the excavated wall of the channel (Murthy 1980).

1.2 Bye-pass tunnel

Within two months of the commissioning of the project, a portion of head race tunnel which was passing through a low cover reach, in a bedded shale and sandstone formation collapsed following heavy rains and massive landslides. The powerhouse was shut down and emergency gates at intake were lowered to dewater the tunnel. The settlement was of the order of 3 to 5m (Divatia and Taneja 1987). It was found that the steel liner provided during construc-

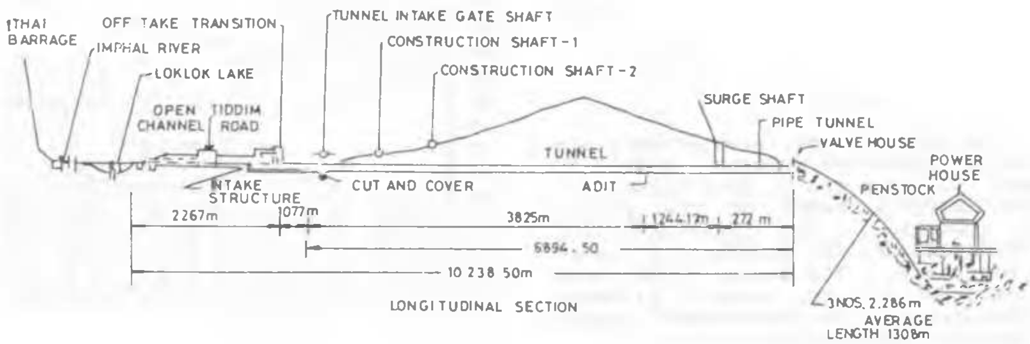

Figure 1. Longitudinal section of general scheme of Loktak hydro-electric project.

tion of the tunnel in the low cover reach had bent, folded and buckled in the direction of the slide. In order to re-commission the project, a bye-pass steel lined tunnel was constructed on the uphill side of the collapsed tunnel. The construction of bye-pass tunnel and placement of steel liner delayed the power generation for about a year. In addition to constructing the bye-pass tunnel, it was essential to carry out some surface protection works to provide surface drainage and to stop further reduction of low cover due to erosion and slides. It was also decided to monitor the stability of these slopes on a long term basis.

1.3 Penstock slopes

The penstocks are supported on 12 anchor blocks (AB) and 68 saddles. The strata particularly between anchor blocks 4 and 5 and 11 and 12 are highly unstable and often been subjected to sloughing resulting in the movement of saddles. The lithology exposed are soft, crumbly shales, siltstones, sandstones and slope wash materials. The shales are carbonaceous.

In the year 1973-74, massive subsidence in the area downstream of AB-5 took place. As a result, the saddle supports between AB-5 and AB-6 moved laterally by about two metres. The design of the saddle supports was reviewed, and saddle cum retaining walls were constructed with the foundations going upto rock. Boulder filling was done to make up the remaining gap. In a subsequent slip, this boulder filling collapsed. The new stabilization measures included vertical shafts going to a depth of 12 to 15m, which provided drainage as well as shear resistance.The top of the shafts were tied by reinforced concrete ribs. A number of horizontal holes 30 to 40m deep were drilled for drainage. Three shafts were driven at an angle of 45° to penstock alignment to relieve pressure on ground. These measures helped in controlling the movements but did not stop them.

Two rows of bored piles were driven into the rock and tied at top. Pumps were installed in the shafts for improving drainage (Madhavan 1988).

Another major slide took place in October, 1986. Fortunately, no major project work was affected due to this slide except some residential buildings just above the surge shaft.

The effectiveness of the above stabilisation measures to arrest slope movements was monitored. The inclinometer systems installed for monitoring hill slopes in bye-pass tunnel and penstock area are detailed below.

2 THE INCLINOMETER SYSTEM

The biaxial type model of inclinometer manufactured by slope Indicator Company (model SI 1000) was used. It consists of four major components: the borehole sensor or the probe, the digital indicator, the interconnecting cable, and the guide casings permanently installed inside boreholes. The casing pipes have an outer diameter of 69.8 mm, internal diameter of 58.9 mm and are of 1.5m length each, which are connected with each other using couplings.

3 THE INSTALLATION

A total number of four holes were fitted with Slope Indicator Company (SINCO) casings during the month of January-February, 1986 on the slopes in between anchor blocks 11 and 12 of Penstock area as indicated in location plan (Figure 2). For this purpose, holes of 150mm diameter were drilled and the plastic casings of 69.8mm diameter with four longitudinal grooves equally spaced around the inside circumference were grounded inside the boreholes using cement slurry. The casings were extended up to a depth of about 3m inside bed rock as shown in Figure 3 of geological section along BC (Figure 2). A cap was fitted at the bottom of the casings. The four directions of monitoring were marked on the casing as A+A- and B+B-. The casings were considered ready for monitoring after the grout was set.

4 MONITORING

During observations, the inclinometer probe was lowered into the borehole to the bottom of the casing. It was then gradually pulled up stopping at intervals of every 0.5 metre for taking observations. Readings from the readout were recorded both in the A+A- and B+B- direction, corresponding to the direction of slope and normal to the same respectively. The

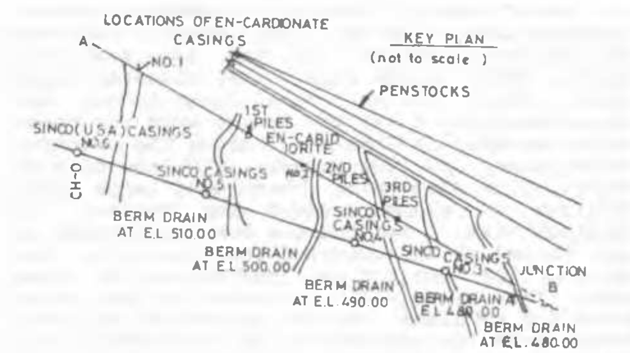

Figure 2. Key plan for inclinometer casings in penstock area.

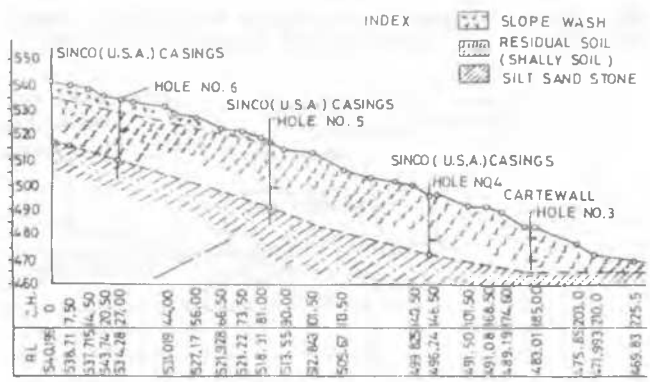

Figure 3. Geological section along BC.

initial readings were taken using the grooves in both directions. Since all inclinometer readings are referenced to an original set of measurements, extreme care was exercised during obtaining the initial set of observations. The measurements of original casing profile were repeated three times for confirming the accuracy of the reading and as well as the probe and its reproducibility.

5 RESULTS AND DISCUSSIONS

A total of eight sets of measurements including initial set covering over a period of 2 years have been presented in this paper. The SINCO casings were installed in boreholes numbers 3 to 6 in Penstock area between anchor blocks 11 and 12 as shown in key plan (Figure 2). The geological section along BC is shown in Figure 3 along with depth and position of the casings.

Figures 4 to 7 show the resultant cumulative displacement recorded at various depths in boreholes 3,4,5 and 6 respectively. The displacements have been computed based on the difference between the present and initial set of readings for two mutually perpendicular directions. The resultant displacements have been arrived at by summing up vectorially the displacements recorded in two directions. The resultant displacements have been added up from the bottom to the top of the casing for getting the cumulative movement. The inclinometer probe got stuck at 4m depth in borehole 3 (the casing nearest to the powerhouse) indicating significant movement arround this depth. During the earlier observations, the maxiumum movements in this casing were noted at the depth of 4.5m and 12.5m which apparently has prevented the probe from going down. A small slip on the surface had also occurred in this area during the rainy season of 1987. The data obtained from this casing are shown in Figure 4 while Figures 5 to 7 show these obtained from other three casings.

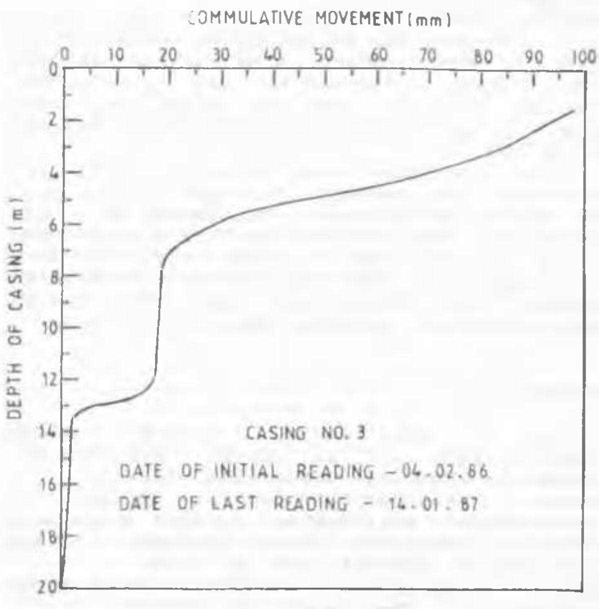

Figure 4. Cummulative movement versus depth of casing for borehole 3.

The net displacements at specific depths have been arrived at by the two displacement vectors from A+A- and B+B- directions. Figure 8 shows the variation of the net displacement with respect to time which shows that the displacements have increased with time. The maximum movement recorded over a period of 2 years (excluding hole 3 due to failure) is about 24mm which occurred at a depth of 12m in casing no. 6. The depths at which maximum displacement has taken place, have been marked in Figure 3.

The net cumulative displacements with respect to time have been shown in Figure 9. The maximum movement of the order of 103 mm has been noted in casing 3. It is also seen from the observation that the least movement is noted in casing 6.

From the observations of the inclinometer system carried out by CSMRS, it was established that there are positive indications of movement in the penstock area. The maximum displacement at a particular depth was of the order of 25mm and a cylindrical surface appeared to be developing. The rate of movement over the period also increased. Corrective measures were therefore called for to reduce further movements.

6 CONCLUSIONS

1. Stability of slopes in water resources development projects pose serious problems. Adequate site investigations are essential for arriving at final locations and alignments of

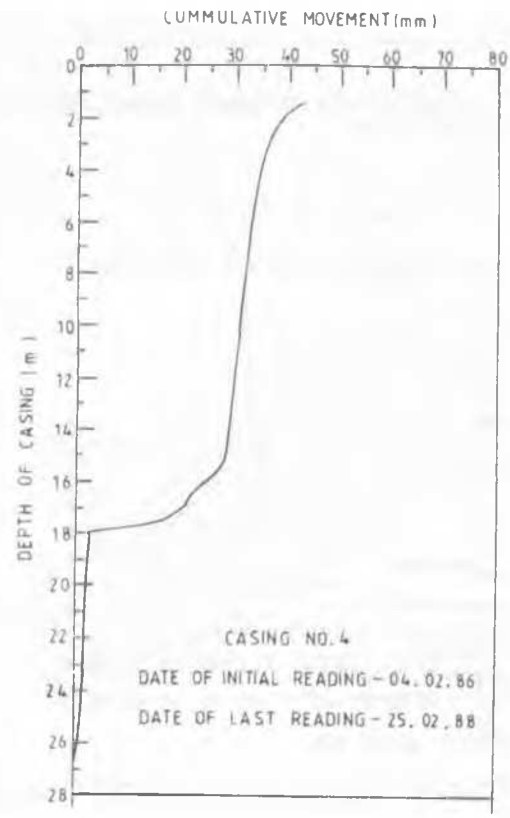

Figure 5. Cummulative movement versus depth of casing for borehole 4.

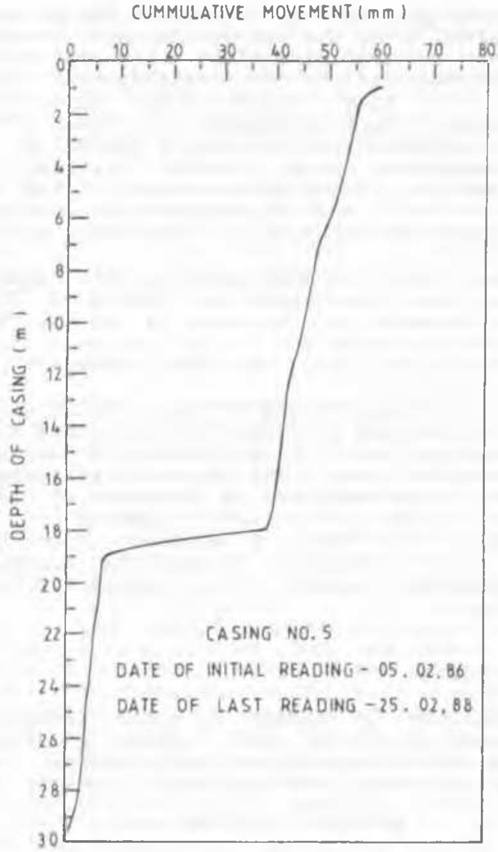

Figure 6. Cummulative movement versus depth of casing for borehole 5.

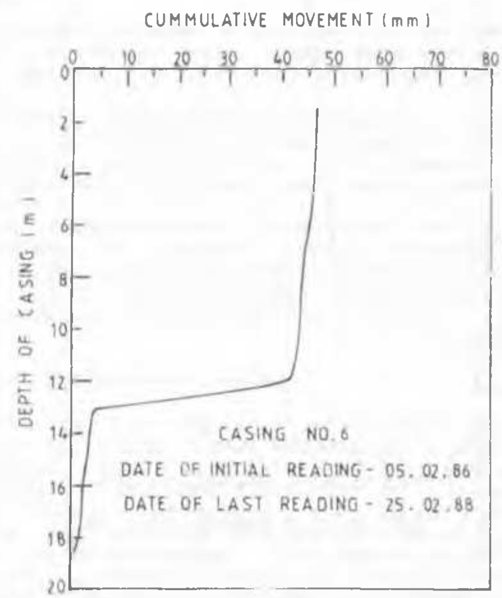

Figure 7. Cummulative movement versus depth of casing for borehole 6.

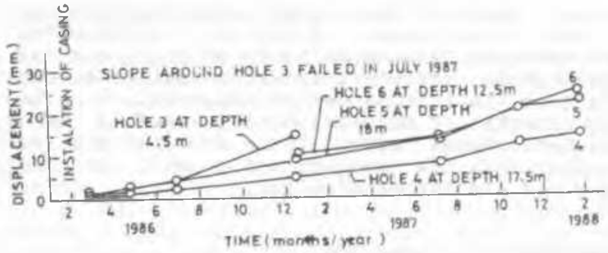

Figure 8. Displacement versus time plot for all casings.

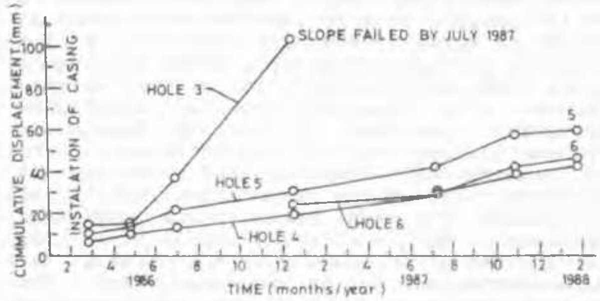

Figure 9. Cummulative displacement versus time plot for all casings.

various appurtenant works. It is needless to stress on this aspect particularly for important projects located in the north-eastern Himalayan ranges where the hills comprise of deeply weathered shales, siltstones and sandstones and valleys covered with slope wash material.

2. At Loktak Project, problems were encountered during excavation of open channel, which had to be stabilized with sheet piles. A landslide caused failure of head race tunnel and a bye-pass tunnel had to be constructed. Slopes in the penstock area failed several times during construction and afterwards. Elaborate remedial measures comprising deep shafts, piles surface and sub-surface drainage were called for.

3. The programme undertaken by CSMRS for monitoring the movement of slopes in the bye-pass tunnel and penstock area though on a limited scale, has established that the movement of slope is still taking place. A slip surface is developing and that the rate of movement has increased over the period, and that future urgent corrective measures were called for.

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